Missouri Department of Transportation

Bridge Division

Bridge Design Manual

Section 3.40

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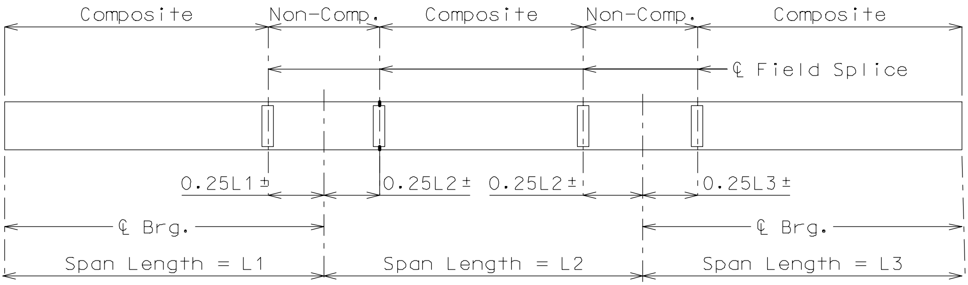
GENERAL

This section pertains to structures composed of steel girders with concrete slab connected by shear connectors.

The stresses of composite girders and slab shall be computed based on the composite cross-section properties and shall be consistent with the properties of the various materials used.

The regions subjected to positive moment are considered as composite and the regions subjected to negative moment are considered as non-composite.

For the initial girder design, composite/non-composite regions can be approximately assumed as:



SECTION PROPERTIES

Cross-section properties of the composite section shall include concrete slab and steel section.

Cross-section properties of the non-composite section shall include steel section only.

Use composite property for positive moment section.

Use non-composite property for negative moment section. The effect of reinforcing steel in the section is not considered.

The ratio of modulus of elasticity of steel to that of concrete, n, shall be assumed to be eight. The effect of creep shall be considered in the design of composite girders which have dead loads acting on the composite section. In such structures, n=24 shall be used.

DESIGN UNIT STRESSES (also see note A1.1 in Section 4)

Reinforcement Concrete

Reinforcing Steel (Grade 60)	fs = 24,000 psi	fy = 60,000 psi
Class B2 Concrete (Superstructure)	fc = 1,600 psi	f'c = 4,000 psi

Structural Steel

Structural Carbon Steel (ASTM A709 Grade 36)	fs = 20,000 psi	fy = 36,000 psi
Structural Steel (ASTM A709 Grade 50)	fs = 27,000 psi	fy = 50,000 psi
Structural Steel (ASTM A709 Grade 50W)	fs = 27,000 psi	fy = 50,000 psi

SHEAR CONNECTORS DESIGN

The shear connectors shall be designed for fatigue and checked for ultimate strength (AASHTO Article 10.38.5.1).

Step 1:

Compute Vr, the range of shear in kips, from the structural analysis, due to live loads and impact, for entire span.

At any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads).

Step 2:

Use the average Vr per span, for the section of the span that is assumed to act compositely (from end of span to point of contraflexure for end spans, or from point of contraflexure to point of contraflexure for int. spans).

Step 3:

Using the average Vr from step 2, compute the range of horizontal shear load per linear inch, Sr in kips per inch, at the junction of the slab and stringer from the following equation:

$$Sr = \frac{VrQ}{I} \quad \text{(AASHTO Article 10.38.5.1.1 Eq. 10-58)}$$

where:

Q = static moment of the transformed compressive concrete area about the neutral axis of the composite section, in cubic inches (✳);

I = moment of inertia of the transformed composite girder in positive moment regions in inches to the fourth power (✳).

✳ In the formula, the compressive concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modula ratio n=8.

Step 4:

Compute the allowable range of horizontal shear, Zr, in pounds on an individual connector, welded stud, by use of the following formula:

$$Zr = \alpha d^2 \quad \text{(AASHTO Article 10.38.5.1.1 Eq. 10-60)}$$

where:

$$\frac{H}{d} \geq 4$$

H = height of stud in inches;

d = diameter of stud in inches;

α = 13,000 for 100,000 cycles
10,600 for 500,000 cycles
7,850 for 2,000,000 cycles
5,500 for over 2,000,000 cycles.

SHEAR CONNECTORS DESIGN (CONT.)

Step 5:

Compute the required pitch of the shear connector units, pitch by the following formula:

$$\text{Pitch} = \frac{\Sigma Z_r}{S_r}$$

Where:

Pitch = required pitch, in inches;

ΣZ_r = the resistance of all connectors at one (1) transverse girder cross-section as a shear connector unit.

Note:
The pitch is to be constant and spaced in the composite section (round to the nearest inch).

Step 6:

Compute the number of additional connectors required at point of contraflexure, N_c, from the following formula:

$$N_c = \frac{A_r^s f_r}{Z_r} \qquad \text{(AASHTO Article 10.38.5.1.1 Eq. 10-69)}$$

where:

N_c = number of additional connectors required at the point of contraflexure;

A_r^s = total area of longitudinal slab reinforcing steel for each girder over interior support;

f_r = range of stress due to live load plus impact, in the slab reinforcement over the support (in lieu of more accurate computations, f_r may be taken as equal to 10,000 psi);

Z_r = the allowable range of horizontal shear on an individual connector.

This number of additional connectors, N_c, shall be placed adjacent to the point of dead load contraflexure within a distance equal to 1/3 of the effective slab width, if it is possible. If it is impossible, use minimum pitch of 6".

SHEAR CONNECTORS DESIGN (CONT.)

Step 7: Check connectors for ultimate strength

The number of connectors provided for fatigue must be checked to ensure that adequate connectors are provided for ultimate strength.

To check for ultimate strength, proceed as follows:

(1) Compute the force in the slab (P), which is defined as: at the point of maximum positive moment, the force in the slab is taken as the smaller value of the following two formulae:

$$P_1 = A_s F_y \quad \text{(AASHTO Article 10.38.5.1.2 Eq. 10-62)}$$

or

$$P_2 = 0.85 f'_c b t_s \quad \text{(AASHTO Article 10.38.5.1.2 Eq. 10-63)}$$

Where:

A_s = total area of the steel section including cover plates (if used);

F_y = specified minimum yield point of the steel being used;

f'_c = compressive strength of concrete at age of 28 days;

b = effective flange width given in AASHTO Article 10.38.3;

t_s = thickness of concrete slab.

SHEAR CONNECTORS DESIGN (CONT.)

Step 7: Check connectors for ultimate strength (continued)

(2) Compute the ultimate strength of individual shear connectors, S_u, in pounds, from the following formula:

Welded Studs (for H/d ≥ 4)

$$S_u = 0.4\ d^2 \sqrt{f'_c \bullet E_c}$$ (AASHTO Article 10.38.5.1.2 Eq. 10-67)

Where:

H = height of stud in inches;

d = diameter of stud in inches;

E_c = modulus of elasticity of the concrete, in psi;

$$E_c = 33\ w^{3/2} \sqrt{f'_c}$$ (AASHTO Article 10.38.5.1.2 Eq. 10-68)

w = unit weight of concrete = 150 lb/cu.ft.

(3) Compute the number of connectors between points of maximum positive moment and adjacent end supports, N, (✳), from the following formula:

$$N = \frac{P}{\varnothing S_u}$$ (AASHTO Article 10.38.5.1.2 Eq. 10-61)

Where:

Ø = a reduction factor = 0.85

P = force in the slab as P1 or P2, see page 2.1-3.

✳ Use 2N per span.

Note:

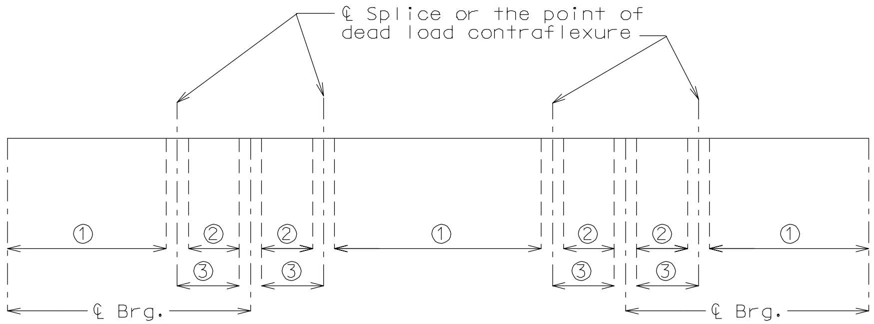
If it becomes impractical to place the number of shear connectors required by ultimate strength in the specified distance (structures with span ratios greater than 1.5); base the number and spacing of shear connectors on the fatigue analysis only.

Increase the haunch by 1/2"± more than what is required to make one size shear connector work for C.I.P. or S.I.P. option.

SHEAR CONNECTORS SPACING

Shear connectors shall not be used on the flange splice plates. Maintain at least 3″ from end of splice plate to nearest connector for fabrication purposes.

For a typical 3-spans bridge, the shear connector units can be approximately arranged as below:



1. Shear connector units at pitch as indicated by design, providing total shear connectors satisfy both fatigue stress and ultimate strength.

2. Provide additional shear connectors to satisfy N_C (see Step 6). Transverse spacing may be different than as in 1 to allow more shear connectors per unit. Minimum of 4 units at 6″ cts.

3. Make the distance of 3 ≤ 1/3 of the effective slab width, if it is possible. If it is impossible, use minimum pitch of 6″ for additional shear studs.

SHEAR CONNECTORS DESIGN EXAMPLE

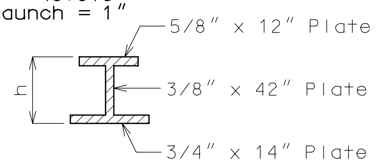
GIVEN:

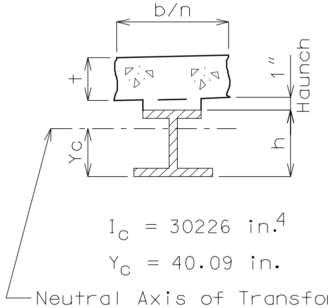
(60'-70'-60')Continuous Composite Plate Girder (ASTM A709 Grade 36 steel)

HS20-44 Loading, Fatigue Case II, Vr(average) = 40.2 kips (✳),

Roadway = 36'-10", Girder spacing = 8'-4", Slab thickness = 8.5".
b = effective slab width = 90", t = effective slab thickness = 7.5",
n = ratio of modulus of elasticity of steel to that of concrete = 8.

Positive Girder Section (✳)
As(girder) = 33.75 sq. in.
h = 43.375"
haunch = 1"





I_c = 30226 in.^{4}

Y_c = 40.09 in.

Neutral Axis of Transformed Composite Section

✳ To simplify this example, the girder cross-section and Vr (average) in positive moment region are assumed equal for all spans.

SOLVE

RANGE OF HORIZONTAL SHEAR LOAD:

$$Q = \text{Statical Moment} = \left(\frac{b \times t}{n}\right)(h - Yc + \text{haunch} + t/2)$$

$$Q = \frac{90''\times 7.5''}{8}(43.375'' - 40\ 09'' + 1'' + 7.5''/2) = 678 \text{ in.}^3$$

$$S_r = \frac{V_r Q}{I} = \frac{40.2 \text{ kips} \times 678 \text{ in.}^3}{30226 \text{ in.}^4} = 902 \text{ lb /in.}$$

ALLOWABLE RANGE OF HORIZONTAL SHEAR ON AN INDIVIDUAL CONNECTOR:

Try d = 3/4", H= 4" Studs; $\frac{H}{d} = \frac{4}{3/4} = 5.33 > 4$; O.K.

Fatigue Case II, Longitudinal member,
Truck loading, Stress Cycle = 500,000,
Lane loading, Stress Cycle = 100,000,

Use Stress Cycle = 500,000 (AASHTO Table 10.3.2A)

α = 10,600 (See page 2.1-1)

$$Z_r = \alpha d^2 = 10,600\ (3/4)^2 = 5962 \text{ lb}$$

REQUIRED PITCH OF SHEAR CONNECTOR UNITS:

Try two shear studs per unit,

$$\text{Pitch} = \frac{\Sigma Z_r}{S_r} = \frac{2 \times 5962 \text{ lb}}{902 \text{ lb/in.}} = 13.2''$$ .... Use 13"

Try 2 shear connectors/unit @ 13" cts-cts for Fatigue Design.

SHEAR CONNECTORS DESIGN EXAMPLE (CONT.)

COMPUTE NUMBER OF ADDITIONAL S.C. REQUIRED @ POINTS OF CONTRAFLEXURE:

Z_r = 5962 lb (from page 2.3-1)

f_r = 10,000 psi (from page 2.1-2)

Use Smaller:

A^s_r = effective slab steel for the girder

= (b=90"/spa=5") x 0.44 sq.in. = 7.92 sq.in. ....USE
(Assume Panel Option: #6 top bars @ 5", eff. slab width=90")

A^s_r = 1% of gross concrete area

$$= \frac{0.01 \times 39.5'(\text{out-out slab}) \times 8.5''(\text{slab thickness})}{5 \text{ Girders}}$$

= 8.06 sq. in.

$$N_c = \frac{A^s_r f_r}{Z_r} = \frac{7.92 \times 10{,}000}{5962} = 13.3; \text{ (Say 14 Shear Connectors)}$$

Therefore, the additional 14 shear connectors are required to be placed adjacent to the point of DL Contraflexure. If a splice plate is located at the point of dead load contrafleflexure, follow the criteria shown on page 2.2-1.

Assume top flange width = 14" for negative girder section, use 5 units (3 studs per unit) at pitch of 6" to provide 15 shear connector studs.

CHECK NUMBER OF S.C. FOR ULTIMATE STRENGTH:

Use Smaller:

$P_1 = A_s F_y$ = 33.75 sq.in. x 36,000 psi = 1,215,000 lb........USE

$P_2 = 0.85\ f'_c\, b\, t$ = 0.85 x 4000 psi x 90" x 7.5" = 2,295,000 lb

Therefore, use $P = P_1$ = 1,215,000 lb

$E_c = 33 \times (150)^{3/2}\sqrt{4000}$ = 3,834,253 psi (See page 2.1-4)

$S_u = 0.4\ d^2 \sqrt{f'_c \times E_c} = 0.4 \times (3/4)^2 \sqrt{4000 \times 3{,}834{,}253}$

$= 0.225 \times 123{,}842.7 = 27{,}865$ lb

$\varnothing = 0.85$

$$N = \frac{P}{\varnothing S_u} = \frac{1{,}215{,}000}{0.85 \times 27{,}865}$$ = 51.3 (Number of shear connectors required between the point of maximum positive moment and the end of support or dead load points of contraflexure by Ultimate Strength.)

NOTE:

Studs/Span may be distributed by average of uniform spacing over entire positive moment region.

Studs/Span = 2N = 2 x 51.3 = 102.6 or 103 studs required.

Try 52 unit x 2 S.C./unit, total = 104 studs for Ultimate Strength.

SHEAR CONNECTORS DESIGN EXAMPLE(CONT.)

CHECK NUMBER OF S.C. FOR ULTIMATE STRENGTH(CONT.):

Shear connector pitch based on ultimate strength

Spans 1 & 3 - (43.5′ ✳)

$$\text{Pitch} = \frac{43.5 \times 12''}{(104/2) - 1} = 10.2'',\ \text{use } 10'' < 13''$$

∴ Ultimate Strength Governs

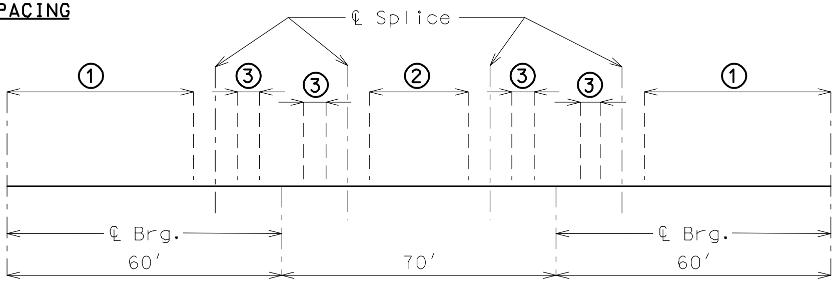
Spans 2 - (36′ ✳)

$$\text{Pitch} = \frac{36 \times 12''}{(104/2) - 1} = 8.5'',\ \text{use } 8'' < 13''$$

∴ Ultimate Strength Governs

✳ Distance where shear connectors can be placed.

S.C. SPACING



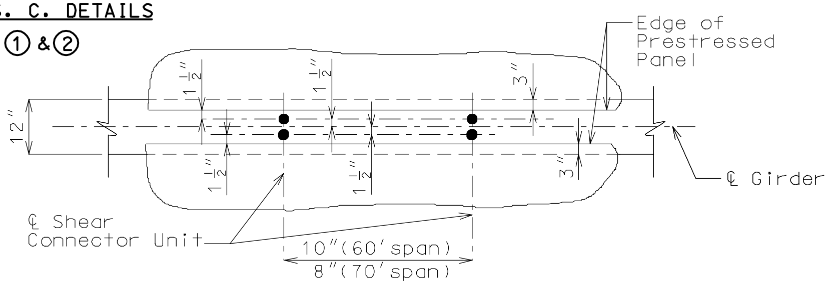
① = 42′-6″, 51 spa. @ 10″ (52 units x 2 S.C./unit = 104 S.C. > 103, OK)

② = 34′-0″, 51 spa. @ 8″ (52 units x 2 S.C./unit = 104 S.C. > 103, OK)

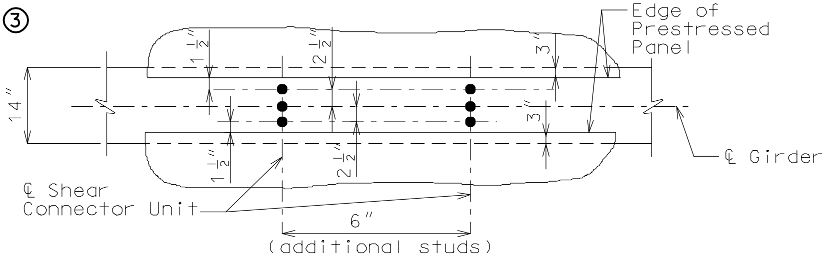
③ = 2′-0″, 4 spa. @ 6″ (5 units x 3 S.C./unit = 15 S.C. > 14, OK)

S. C. DETAILS

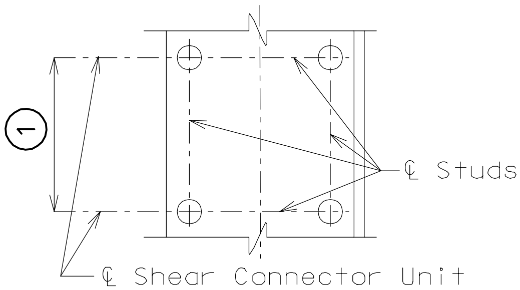
① & ②



③



SHEAR CONNECTOR DETAILS



PLAN

(1) 12″ cts. preferred min., 6″ cts. absolute min., 24″ cts. maximum. (1″ increments min.)

(2) = 2″ minimum for CIP slab;

(2) = a minimum height equal to the top of panel for P/S panel option.

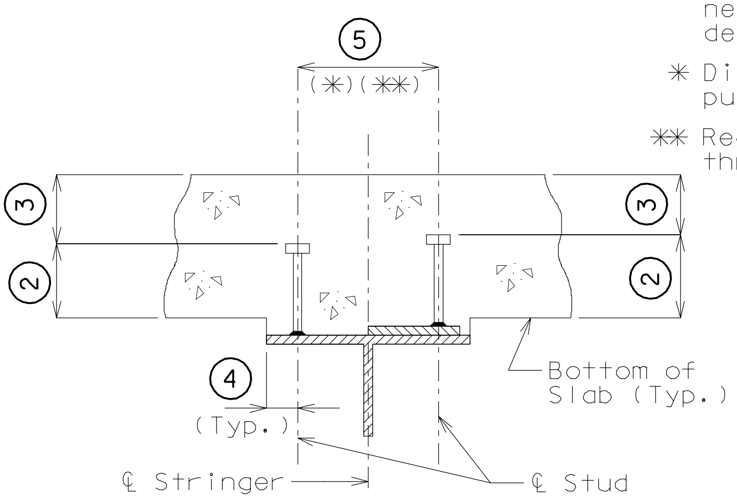
(3) 3″ min. clear depth of concrete cover over shear connectors.

(4) 1-1/2″ (Min.) CIP slab (*)
4-1/4″ (Min.) panel option (*)

(5) 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2-1/4″ absolute minimum.

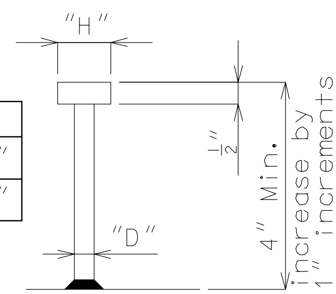
* Dimensions are not for detailing purposes.

** Requires the same dimension throughout the span.



SECTION THRU STRINGER OR GIRDER

"D"	"H"
3/4″	1-1/4″
7/8″	1-3/8″



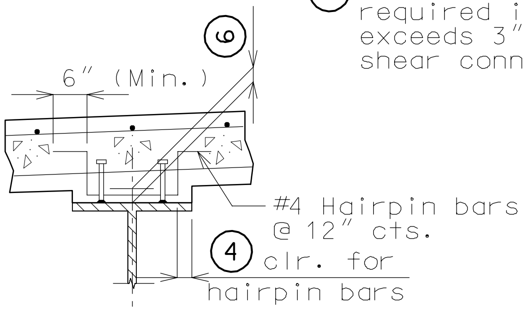
SECTION THRU COVER PLATE
(If required by design)

Quc - USEFUL CAPACITY OF ONE STUD CONNECTOR			WEIGHT IN PLACE PER 100 STUDS			
STUD DIA.	POUNDS	f′c	4″	5″	6″	7″
3/4″	11739	4000	63.0	75.5	88.0	100.5
7/8″	15977	4000	81.0	98.0	115.0	132.0

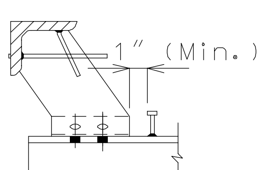
Working capacity = Quc/FS

The above table is based on shear connectors 4″ or greater in length.

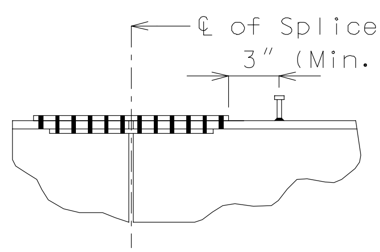
(6) #4 Hairpin bars required if depth exceeds 3″ for 5″ shear connectors.



SECTION THRU HAUNCH

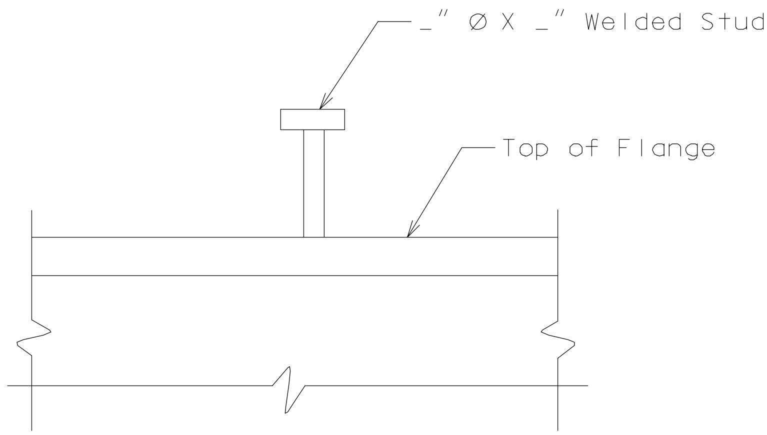


CLEARANCE AT EXPANSION DEVICE

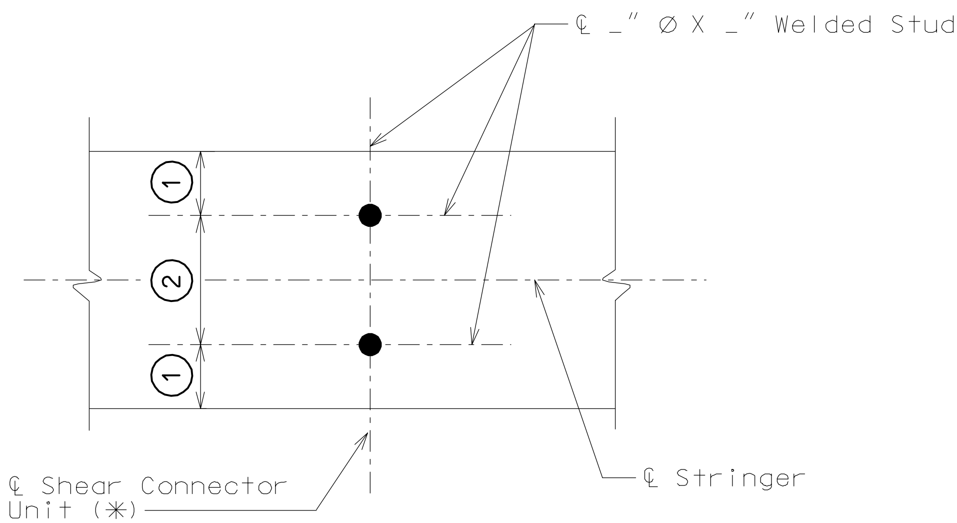


CLEARANCE AT SPLICE PLATE

SHEAR CONNECTOR UNIT



ELEVATION



PLAN

1 = 1-1/2" (Min.) for CIP slab,

1 = 4-1/4" (Min.) for Panel option

2 4 x (Stud diameter) preferred mininmum, may be reduced if necessary for a more economical design; 2-1/4" absolute minimum.

✳ Two studs per unit shown is for illustration purpose only.

Precast Prestressed Concrete Panels on Steel Structures

Use precast prestressed panels on all tangent steel structures. Evaluate the viability of the use of P/S panels on curved structures on a case by case basis and use or include as an option to a CIP slab where deemed appropriate.

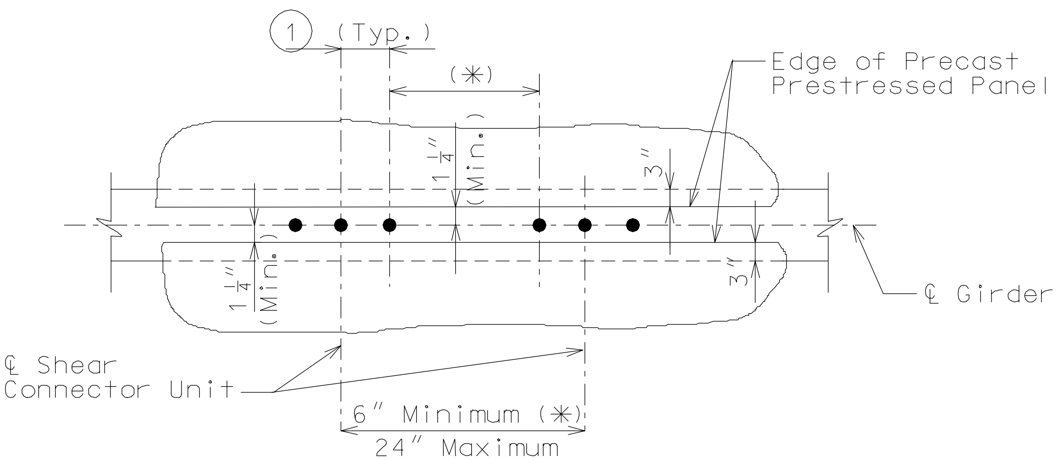
Whenever panels are used, the minimum top flange width shall be 12" for Plate Girders and 10" for Wide Flange Beams.

Steel girders shall be cambered when using P/S Panels. Minimum joint filler thickness is 3/4", except over splice plates, in which case use 1/4" minimum. Maximum joint filler thickness is 2".

Shear connectors shall have a minimum height equal to the top of panel.

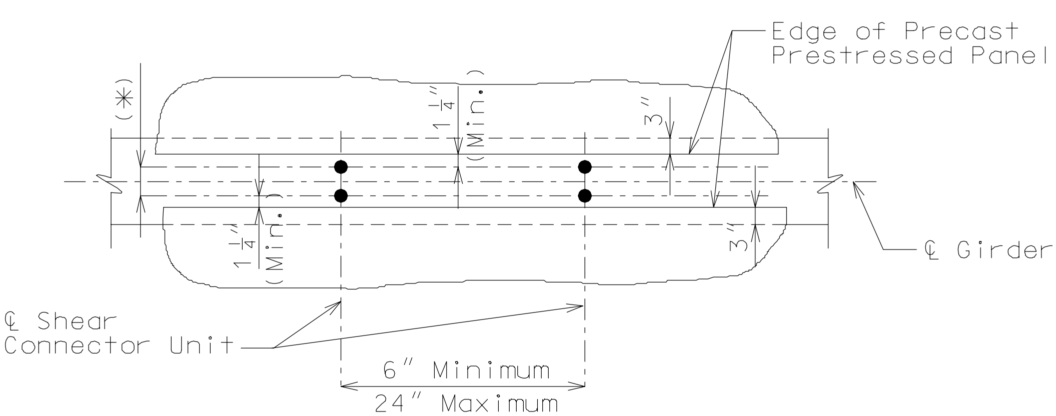
Shear connectors shall be spaced by units and shear connectors in each unit shall be placed along ℄ of girder. On wide flange widths, two lines of connectors may be used if spacings and clearances allow.

Additional shear connectors, Nc, at point of contraflexure may be placed in units normal to ℄ girder as space allows or in a single row along ℄ girder as shown below:



* Adjacent shear connectors shall not be closer than (1) center to center, either longitudinal or transversely.

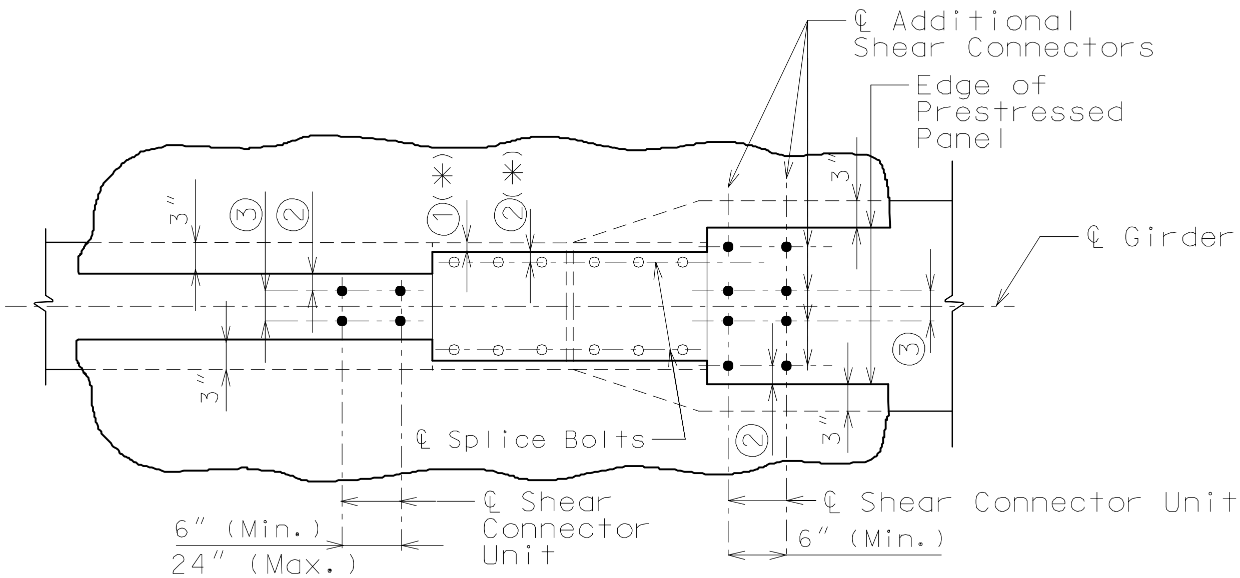
(1) 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2-1/4" absolute minimum.



P/S strands shall extend 3" minimum and 6" maximum past edge of precast prestressed panel and not closer than 1" to the adjacent panels.

PRECAST PRESTRESSED CONCRETE PANELS ON STEEL STRUCTURES (CONT.)

Panel end at splices shall be notched to avoid bolt heads as shown below:



① 3/4" min. wide bearing edge for panel at splice, typ. (✳)

② 1-1/4" min. (Typ.)

③ 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2-1/4" absolute minimum.

✳ In order to meet ① and ② above, it is necessary to have an edge bolt distance of 2" or greater for splice plate. For unusual cases, which would require field splices for flange widths 14" or 15" for P/S precast panel option, it will be necessary to change the top flange width to either 13" or 16" of equal area to maintain the 3/4" minimum panel bearing edge on the splice plates.

Minimum joint filler thickness is 3/4" except over splice plates in which case use 1/4" minimum. When joint filler is less than 1/2" thick over splice plate, make the width of joint filler at splice the same width as panel on splice (maximum 1-1/2" wide).

Maximum difference in top of flange thickness should be checked so that joint filler thickness does not exceed 2".

DEFLECTION

Allowable Live Load Deflection

1. Composite Design: Defl. = 1/1000 of span:
2. Non-Composite Design: Defl. = 1/800 of span.

   Where:

   Defl. = allowable deflection due to service live load plus impact.

Dead Load Deflection

Compute at 1/4 point for bridge with spans less than 75', at 1/10 points for spans 75' and over.